IRC : 45-1972

RECOMMENDATIONS FOR ESTIMATING THE RESISTANCE OF SOIL BELOW THE MAXIMUM SCOUR LEVEL IN THE DESIGN OF WELL FOUNDATIONS OF BRIDGES

THE INDIAN ROADS CONGRESS
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Published by

THE INDIAN ROADS CONGRESS

Jamnagar House, Shahjahan Road

New Delhi-11

1996

Price Rs. 80/-

(Plus Packing & Postage)

IRC : 45-1972

First Published : October, 1972
Reprinted : March, 1984
Reprinted : July, 1987
Reprinted : March, 1992
Reprinted : August, 1996
Reprinted : November, 2002
Reprinted : January, 2006
Reprinted : June, 2009

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Printed by Abhinav Prints, Delhi-110041
(500 Copies)

RECOMMENDATIONS FOR ESTIMATING THE RESISTANCE OF SOIL BELOW THE MAXIMUM SCOUR LEVEL IN THE DESIGN OF WELL FOUNDATIONS OF BRIDGES

1. INTRODUCTION

1.1. The draft recommendations for estimating the resistance of soil below the maximum scour level in the design of well foundations of bridges were finalised by a Subcommittee consisting of the following personnel at their meeting held on the 1st March 1971.

1. Shri B. Balwant Rao — *Convenor*
2. Shri S. Seetharaman — *Member-Secretary*

Members

3. Shri S. B. Joshi
4. Dr. R. K. Katti
5. Shri S. M. Kaul
6. Dr. P. Ray Chowdhury
7. Shri N. S. Ramaswamy
8. Dr. K. S. Sankaran
9. Shri Shitala Sharan
10. Shri S. N. Sinha
11. Shri T. N. Subba Rao

This draft was approved by the Bridges Committee in their meetings held on the 17th November, 1971 and 14th April, 1972. It was later approved by the Executive Committee in their meeting held on the 26th and 27th April, 1972 and by the Council in their 78th meeting held in Nainital on the 10th July, 1972.

1.2. The recommendations given in this Standard have been formulated on the basis of the observed behaviour of models of well foundations and also the work done by many workers in this field. The basic assumptions are given in Appendices.

1.3. These studies have indicated that :

(i) sharing of the moment between sides and base is continuously changing with the increase in deformation of the soil : and

(ii) the mechanics of sharing of the moment between the sides and the base is entirely different for the initial stages of loading of a well as compared to its ultimate failure

1.4. Elastic theory method gives the soil pressures at the side and the base under design loads, but to determine the actual factor of safety against failure, it will be necessary to calculate the ultimate soil resistance. Therefore, the design of well foundations shall be checked by both these methods.

2. SCOPE

2.1. The procedure given is applicable to the design of well foundations of bridges resting on non-cohesive soil like sand and surrounded by the same soil below maximum scour level. The provisions of these recommendations will not apply if the depth of embedment is less than 0.5 times the width of foundation in the direction of lateral forces.

3. PROCEDURE FOR CALCULATING THE SOIL RESISTANCE

The resistance of the soil surrounding the well foundation shall be checked :

(i) for calculation of base pressures by the elastic theory with the use of subgrade moduli ; and

(ii) by computing the ultimate soil resistance with appropriate factor of safety.

4. METHOD OF CALCULATION

I. **Elastic Theory** (vide *Annexure* 1)

Step 1 : Determine the values of W, H and M under combination of normal loads without wind and seismic loads assuming the minimum grip length below maximum scour level as required under IRC : 5—1970* :

where

W = total downward load acting at the base of well, including the self weight of well.

* Standard Specifications and Code of Practice for Road Bridges, Section I—General Features of Design.

H = external horizontal force acting on the well at scour level.

M = total applied external moment about the base of well, including those due to tilts and shifts.

Step 2 : Compute I_B and I_V and I ;

where

I = $I_B + mI_V (1 + 2 \mu' \alpha)$

I_B — moment of inertia of base about the axis normal to direction of horizontal forces passing through its C.G.

I_V = moment of inertia of the projected area in elevation of the soil mass offering resistance = $\frac{LD^3}{12}$;

where

L = projected width of the soil mass offering resistance multiplied by appropriate value of shape factor.

Note : The value of shape factor for circular wells shall be taken as 0.9. For square or rectangular wells where the resultant horizontal force acts parallel to a principal axis, the shape factor shall be unity and where the forces are inclined to the principal axis, a suitable shape factor shall be based on experimental results.

D = depth of well below scour level.

m = K_H/K : Ratio of horizontal to vertical coefficient of subgrade reaction at base. In the absence of values for K_H and K determined by field tests m shall generally be assumed as unity.

μ' = coefficient of friction between sides and the soil = tan δ, where δ is the angle of wall friction between well and soil.

$$\alpha = \frac{B}{2D} \text{ for rectangular well}$$

$$= \frac{\text{diameter}}{\pi D} \text{ for circular well}$$

Step 3 : Ensure the following :

$$H > \frac{M}{r}(1 + \mu\mu') - \mu W$$

$$\text{and } H < \frac{M}{r}(1 - \mu\mu') + \mu W$$

where

r = D/2. I/mIv

μ — coefficient of friction between the base and the soil. It shall be taken as tan ϕ.

ϕ = angle of internal friction of soil.

Step 4 : Check the elastic state

$$mM/I \not> \gamma (K_P - K_A)$$

if mM/I is $> \gamma (K_P - K_A)$, find out the grip required by putting the limiting value $mM/I = \gamma (K_P - K_A)$

where

γ = density of the soil (submerged density to be taken when under water or below water table).

K_P & K_A = passive and active pressure coefficients to be calculated using Coulomb's theory, assuming 'δ', the angle of wall friction between well and soil equal to $\frac{2}{3}\phi$ but limited to a value of $22\frac{1}{2}°$.

Step 5 : Calculate

$$\left.\begin{matrix}\sigma_1 \\ \sigma_2\end{matrix}\right\} = \frac{W - \mu' P}{A} \pm \frac{M.B}{2I}$$

where

σ_1 and σ_2 = maximum and minimum base pressure respectively.

A = area of the base of well.

B = width of the base of well in the direction of forces and moments.

P = M/r

Step 6 : Check $\sigma_2 \nless 0$, i.e., no tension

$\sigma_1 \ngtr$ allowable bearing capacity of soil.

Step 7 : If any of the conditions in Steps 3, 4 and 6 or all do not satisfy, redesign the well accordingly.

Step 8 : Repeat the same steps for combination with wind and with seismic case separately.

II. ULTIMATE RESISTANCE METHOD (*Vide Annexure* 2)

Step 1 : Check that $W/A \ngtr \sigma_u/2$

W = total downward load acting at the base of well, including the self weight of well, enhanced by a suitable load factor given vide Step 6.

A = area of the base of well.

σ_u = ultimate bearing capacity of the soil below the base of well.

Step 2 : Calculate the base resisting moment M_b at the plane of rotation by the following formula :

M_b = QWB tan ϕ

B = width in case of square and rectangular wells parallel to direction of forces and diameter for circular wells.

Q = a constant as given in Table I below for square or rectangular base. A shape factor of 0.6 is to be multiplied for wells with circular base.

ϕ = angle of internal friction of soil.

TABLE 1

D/B	0.5	1.0	1.5	2.0	2.5
Q	0.41	0.45	0.50	0.56	0.64

Note : The values of Q for intermediate D/B values in the above range may be linearly interpolated.

$$M_S = 0.10\ \gamma\ D^3\ (K_P - K_A)\ L$$

where

γ = density of soil (submerged density to be taken for soils under water or below water table)

L = projected width of the soil mass offering resistance. In case of circular wells, it shall be 0.9 diameter to account for the shape.

D = depth of grip below maximum scour level.

K_P, K_A = passive and active pressure coefficient to be calculated using Coulomb's theory assuming " δ " angle of wall friction between well and soil equal to $\frac{2}{3}\phi$ but limited to a value of $22\frac{1}{2}°$.

Step 3 : Calculate the resisting moment due to friction at front and back faces (M_f) about the plane of rotation by following formulae :

(i) For rectangular well

$$M_f = 0.18\ \gamma\ (K_P - K_A)\ L.B.D^2 \sin\delta$$

(ii) for circular well

$$M_f = 0.11\,\gamma\,(K_P - K_A)\,B^2D^2 \sin\delta$$

Step 4 : The total resistance moment M_t about the plane of rotation shall be

$$M_t = 0.7\,(M_b + M_S + M_f)$$

Step 5 : Check $M_t \nless M$

where

M = total applied external moment about the plane of rotation, viz., located at 0.2D above the base, taking appropriate load factors as per combinations given below :

$1.1D$ ... (1)

$1.1D + B + 1.4\,(W_C + E_P + W \text{ or } S)$ ... (2)

$1.1D + 1.6L$ ... (3)

$1.1D + B + 1.4\,(L + W_C + E_P)$ ... (4)

$1.1D + B + 1.25\,(L + W_C + E_P + W \text{ or } S)$ ... (5)

where

D = dead load

L = live load including braking, etc.

B = buoyancy

W_C = water current force

E_P = earth pressure

W = wind force

S = seismic force

Note (i) : For horizontal force due to frictional resistance of bearing due to dead and live loads, appropriate factors shall be taken. But effect of deformation due to temperature, shrinkage and creep may be neglected for normal structures.

Note (ii) : Moment due to shift and tilt of wells and piers and direct loads, if any, shall also be considered about the plane of rotation.

Step 6 : If the conditions in Steps 1 and 5 are not satisfied, redesign the well.

ELASTIC THEORY METHOD (*Annexure I*)

1. INTRODUCTION

The following assumptions are made in deriving the equations based on elastic theory :

(i) The soil surrounding the well and below the base is perfectly elastic, homogenous and follows Hooke's Law.

(ii) Under design working loads, the lateral deflections are so small that the unit soil reaction "p" increases linearly with increasing lateral deflection "z" as expressed by $p = K_H z$ where K_H is the coefficient of horizontal subgrade reaction at the base.

(iii) The coefficient of horizontal subgrade reaction increases linearly with depth in the case of cohesionless soils.

(iv) The well is assumed to be a rigid body subjected to an external unidirectional horizontal force H and a moment Mo at scour level.

2. SYMBOLS

A = area of base of the well.

B = width of the base parallel to the direction of the external horizontal force.

D = depth of well below scour level.

H = external horizontal force acting on the well at scour level.

I_B = moment of inertia of the base about an axis passing through C.G. and perpendicular to horizontal resultant force.

I_V = moment of inertia about the horizontal axis passing through the C.G. of the projected area in elevation of the soil mass offering resistance $= \frac{LD^3}{12}$.

K = coefficient of vertical subgrade reaction at the base.

K_H = coefficient of horizontal subgrade reaction at the base.

K_A, K_P = active and passive pressure coefficients for cohesionless soils as per Coulomb's theory.

L = projected width of the soil mass offering resistance.

Note : A shape factor of 0.9 may be applied for circular wells.

m $= \frac{K_H}{K}$, i.e., ratio of the horizontal to the vertical coefficient of subgrade reactions at the base.

M = total applied external moment at the base $= (M_O + H.D)$

M_O = moment of the external forces at scour level.

M_P = moment of P about the base.

M_B = resisting moment at the base.

P = horizontal soil reaction.

μ = coefficient of friction between the base and the soil.

μ' = coefficient of friction between sides and the soil.

γ = density of soil (submerged density to be used when under water)

ϕ = angle of internal friction of soil.

δ = angle of friction between the sides of well and soil taken equal to $\frac{2}{3}\phi$ limited to a value of $22\frac{1}{2}°$.

θ = angular rotation of the well as a rigid body.

σ_x = horizontal soil reaction at depth y from scour level.

σ_y = vertical soil reaction at distance X from C.G. of base.

σ_1, σ_2 = maximum and minimum base pressures.

αD = distance from the axis passing the C.G. of base at which the resultant vertical frictional force on side acts normal to the direction of horizontal force = B/2 in case of rectangular wells or, 0.318 diameter in circular wells.

3. EQUATIONS FOR BASE PRESSURES

In the most general case, the centre of rotation can be above the base at C_1, at the base C_2 or below the base at C_3. It can be easily visualised that the base moves towards the centre of rotation, if the latter lies above the base so that the horizontal frictional force at the base acts in the direction of H. If the point of rotation lies below the base by a similar argument, it is seen that horizontal frictional force at base must be in the opposite sense to H. The maximum frictional force which can develop at the base is μ W. At any particular instant only a fraction of it would be acting. Let it be denoted by $\beta\mu$W where β is a factor always less than one. It is, therefore, clear that before movement takes place β must be between 1 and -1 respectively so that we can write that for point of rotation at the base β must be between the limits -1 to 1. In the particular case of heavy wells met with in actual practice, the point of rotation shall be assumed to be at the base. Let the well rotate about a point C at a horizontal distance Xc from the centre of the well shown in Fig. 1.

P = total horizontal soil reaction from the sides.

M_B = resisting moment at the base.

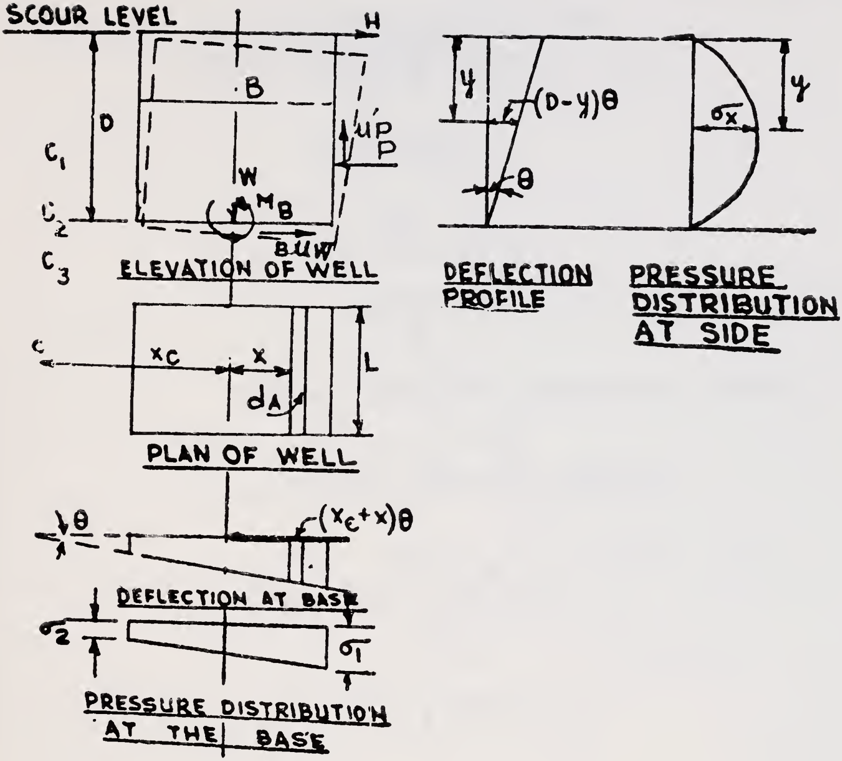


Fig. 1

The total deflection at depth "y" from scour level $= (D - y)\,\theta$

$$\text{Horizontal soil reaction} = \sigma_x = K_H \times \frac{y}{D} \times (D - y)\,\theta$$

$$= m\,\frac{K\,\theta \,.\, y}{D}\,(D - y)$$

Total horizontal soil reaction acting on the sides of the well

$$P = \int_0^D L \,.\, \sigma_x \,.\, d_y$$

$$= m \frac{K\theta . L}{D} \int_{O}^{D} (y D - y^2)\, dy$$

$$= m \frac{K \theta L}{D} . \frac{D^3}{6}$$

$$\text{Putting } \frac{LD^3}{12} = I_v$$

$$P = \frac{2m K \theta I_v}{D} \qquad (1)$$

Let M_P be the moment of P about base level

$$M_P = \int_{O}^{D} \sigma_x (D - y)\, dy . L$$

$$= m \frac{K \theta L}{D} \int_{O}^{D} y (D - y)^2\, dy$$

$$= \frac{m K\theta L}{D} \int_{O}^{D} (yD^2 + y^3 - 2Dy^2)\, dy$$

$$= \frac{m K \theta}{12} . L D^3$$

$$= m K \theta I_v \qquad (2)$$

Now consider the soil reaction acting at the base. Vertical deflection at distance $(X + Xc)$ from centre of rotation $= (Xc + X)\,\theta$

$$\sigma_y = K (X_c + X)\,\theta$$

$$M_B = \int \sigma_y\, d A . X = K \theta \int_{-B/2}^{+B/2} (X_c + X)\, X\, d A$$

$$= K \theta \int_{-B/2}^{+B/2} X^2 dA + K \theta X_c \int_{-B/2}^{+B/2} X\, d A$$

d A being a function of X

As the reference coordinates are at C.G. of base

$$\int X\,d\,A = 0 \text{ and } I_B = \int X^2\,d\,A \quad \text{whence}$$

$$M_B = K\,\theta\,I_B \tag{3}$$

For equilibrium $\Sigma H = 0$

$$H + \beta\mu\,(W - \mu' P) = P$$

$$\text{or } H + \beta\mu W = P\,(1 + \beta\mu\mu')$$

$$\text{or } P = \frac{H + \beta\mu W}{1 + \beta\mu\mu'} \tag{4}$$

Taking moments about base

$$M_O + H\,.\,D = M_B + M_P + \mu' P \alpha D$$

$$\text{or } \quad M = M_B + M_P + \mu' P \alpha D \tag{5}$$

Substituting equations (1), (2) and (3)

$$M = K\,\theta\,I_B + m\,K\,\theta\,I_v + \mu'\alpha\,.\,2\,m\,K\,\theta\,I_v$$

$$= K\,\theta\,[I_B + m\,I_v\,(1 + 2\mu'\alpha)]$$

$$K\,\theta = M/[I_B + m\,I_v\,(1 + 2\mu'\alpha)]$$

$$= \frac{M}{I} \tag{6}$$

where $I = I_B + m\,I_v\,(1 + 2\mu'\alpha)$

From equation (4)

$$\frac{H + \beta\mu W}{1 + \beta\mu\mu'} = P = 2m\,K\,\theta\,I_v/D = 2m\,\frac{M}{I}\cdot\frac{I_v}{D}$$

$$= \frac{M}{r} \quad \text{where } r = \frac{D}{2}\cdot\frac{I}{mI_v}$$

$$H + \beta\mu\,W = \frac{M}{r}\,(1 + \beta\mu\mu')$$

$$\beta\mu\,(W - \frac{M}{r}\,\mu') = \frac{M}{r} - H$$

$$\beta = \frac{\frac{M}{r} - H}{\mu\left(W - \mu'\frac{M}{r}\right)} \tag{7}$$

Equation (7) is satisfied only if $\beta \begin{smallmatrix} < 1 \\ > -1 \end{smallmatrix}$ whence we obtain

$$\frac{M}{r} - H < \mu W - \mu\mu'\frac{M}{r}$$

$$> -\mu W + \mu\mu'\frac{M}{r}$$

$$\text{or } H > \frac{M}{r}(1 + \mu\mu) - \mu W$$

$$< \frac{M}{r}(1 - \mu\mu') + \mu W \tag{8}$$

The vertical soil reaction is given by

$$\sigma_y = K\theta(X_c + X)$$

$$W - \mu' P = \int \sigma_y \, dA = K\theta \int (X_c + X)\, dA$$

$$= K\theta \int X_c \, dA + K\theta \int X \, dA$$

$$= K\theta X_c . A$$

$$\text{whence } X_c K\theta = (W - \mu' P)/A$$

$$\sigma_y = K\theta X_c + K\theta . X$$

$$= \frac{W - \mu' P}{A} + K\theta . X$$

$$\sigma_1 = \frac{W \quad \mu' P}{A} + K\theta . B/2$$

$$\sigma_2 = \frac{W - \mu' P}{A} - K\theta . B/2$$

$$\text{As } K\theta = \frac{M}{I}$$

$$\sigma_1 = \frac{W - \mu' P}{A} + \frac{M \cdot B}{2I}$$

$$\sigma_2 = \frac{W - \mu' P}{A} - \frac{M \cdot B}{2I} \qquad (9)$$

4. CONDITIONS OF STABILITY

(i) The maximum soil reaction from the sides cannot exceed the maximum passive pressure at any depth, if the soil remains in an elastic state. This amounts to the condition that at any depth y

$$\sigma_x = \gamma \,(K_P - K_A)\, y \text{ or}$$

$$m \frac{K\theta}{D} \cdot y\,(D - y) \not> \gamma\,(K_P - K_A)\, y$$

$$\text{or } m \frac{K\theta}{D} (D - y) \not> \gamma\,(K_P - K_A)$$

(at $y = \theta$ L.H.S. is maximum)

$$\text{or } m K\theta \not> \gamma\,(K_P - K_A)$$

$$\text{or } m \frac{M}{I} \not> \gamma\,(K_P - K_A)$$

(ii) The maximum soil pressure at base "σ_1" shall not exceed allowable pressure on soil, similarly the minimum soil pressure "σ_2" shall not be less than 0, i.e., no tension.

ULTIMATE SOIL RESISTANCE METHOD (*Annexure* 2)

1. INTRODUCTION

The elastic theory described in *Annexure* I approximately determines the stresses in the soil mass but does not indicate the safety against ultimate failure of the foundation. For this it will be necessary to know the mode of failure of well foundations.

2. OBSERVED FAILURE OF THE WELL FOUNDATION UNDER ULTIMATE CONDITIONS

The pattern of failure of the soil mass under the application of transverse forces to large and small depths of embedment is depicted in Fig. 2.

Fig. 2

The soil around the base in either case slides over a circular cylindrical path with centre of rotation somewhere above the base. The plastic flow at the side follows the usual concept as in the case of rigid bulkhead at failure. Failure has been observed to occur at about 3° rotation of the well in case of non-cohesive soils.

3. QUANTUM OF RESISTANCE

The observed variation of the total ultimate resistance of the soil mass, i.e., both at the base and the sides under varying direct loads is given in Fig. 3.

This study indicates that the total resisting moment increases with the increase in the ratio of the direct load to the ultimate bearing capacity of the soil up to 0.5 to 0.7. After that it reduces. It is, therefore, necessary to ensure that the bearing pressure adopted has a factor of safety of two or more on ultimate bearing capacity of the soil calculated by any rational formula.

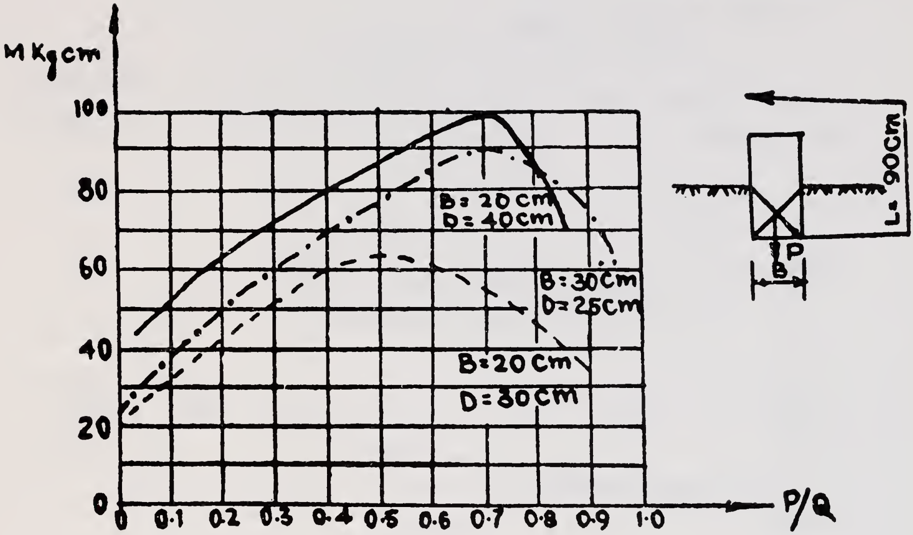


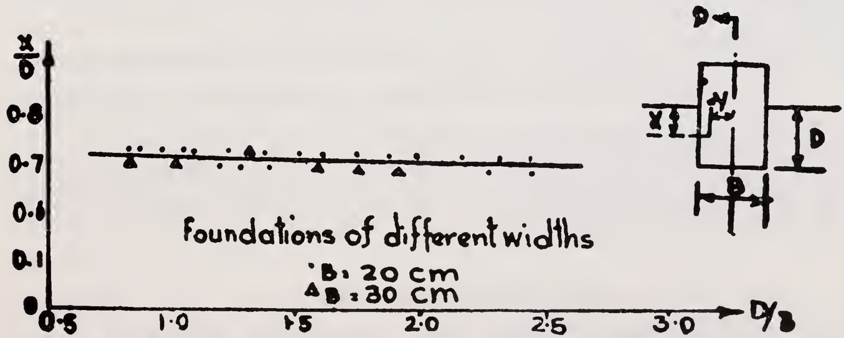
Fig. 3

4. POINT OF ROTATION AT FAILURE

(i) Movement of the point of rotation on the vertical axis

(a) Effect of geometry and horizontal loads

The geometry of the foundation, viz., the ratio of the width of foundation to the depth of embedment in the soil and the magnitude of the horizontal loads have no effect in shifting the point of rotation along the vertical axis as could be seen from Fig. 4.



Position of the centre of rotation as a function of relative depth

Fig. 4

(b) **Effect of direct loads**

The point of rotation has a relation to the ratio of the superimposed vertical loads to the ultimate bearing capacity of the soil as seen from Fig. 5,

where

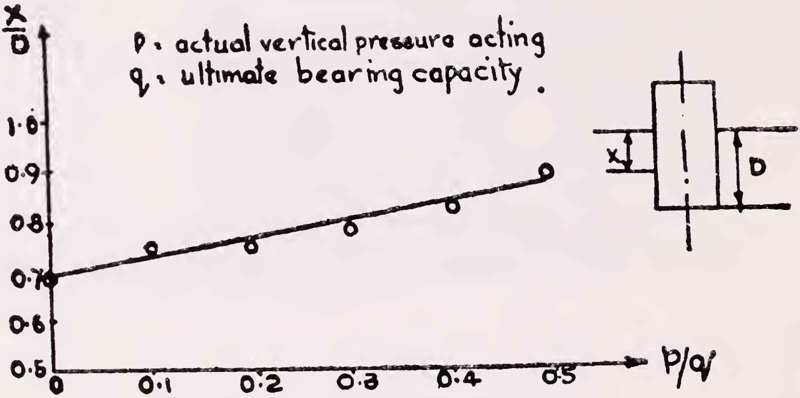


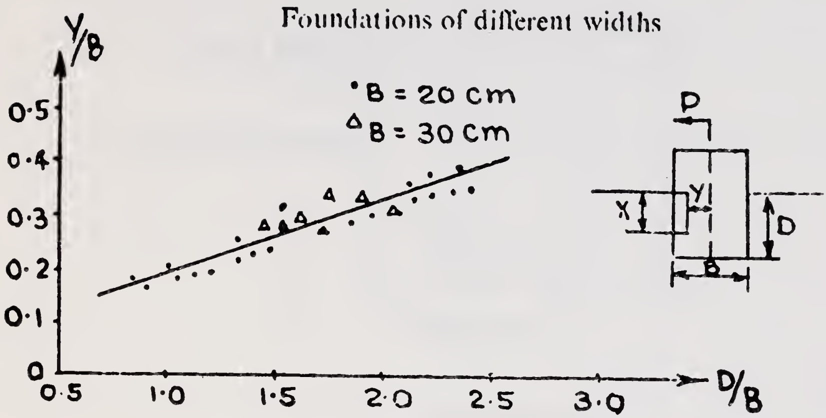
Fig. 5

The actual variation is confined to a narrow range between 0.75 and 0 8 times the depth of embedment below the scour level. Taking into account normally expected vertical loads on well foundations, a fixed value of 0.2 times depth above the base of the foundation has been adopted for working out the soil resistance.

(ii) **Shift of the point of rotation along the horizontal axis**

The point of rotation undergoes a change in the horizontal direction depending upon the geometry of the foundation and the extent of deformation of the foundation. Under ultimate conditions the magnitude of horizontal shift of the point as function of D/B ratio is given in Fig. 6.

This shift in position of the point of rotation in the horizontal direction will cause variation in the share of the moments between the sides and the base.



Position of the centre of rotation as a function of relative depth

Fig. 6

Note : For the purpose of this analysis the shift of the point of rotation along the horizontal axis has been ignored, in view of other related indeterminate factors.

5. METHOD OF CALCULATION

5.1. Base Resisting Moment (M_b)

The base resisting moment is the moment of the frictional force mobilised along the surface of rupture which is assumed to be cylindrical passing through the corners of the base for a square well as shown in Fig. 7. For circular wells, the surface of rupture corresponds to that of a part of sphere with its centre at the point of rotation and passing through the periphery of the base.

If W is the total vertical load augmented by appropriate load factors given in sup-para 5.5 below, the load per unit width will be W/B, which will also be equal to the upward pressure as shown in Fig. 8.

(i) For a rectangular base

Consider the small arc of length $Rd\alpha$ at an angle of α from the vertical axis.

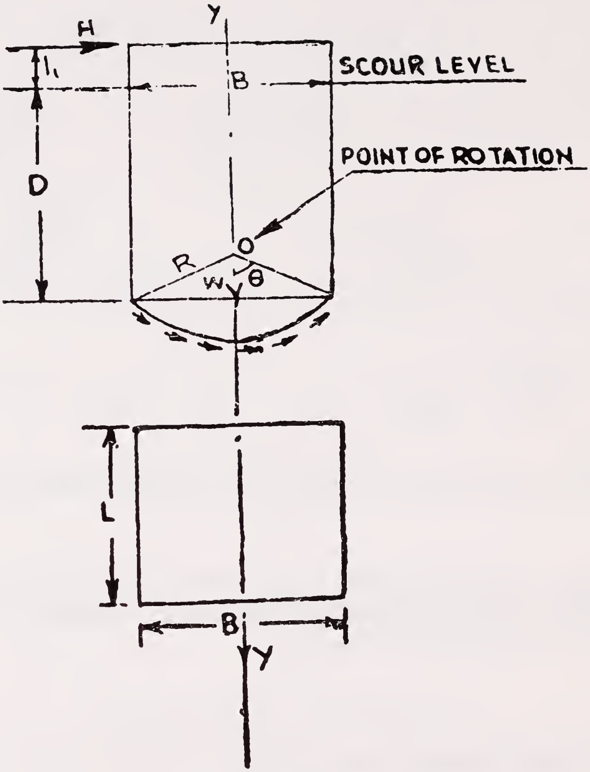
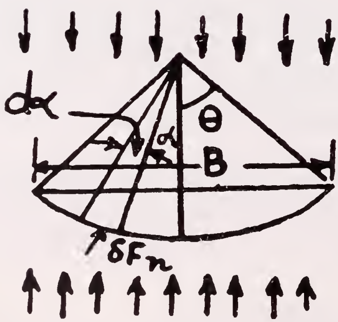


Fig. 7



Applied load W/B for unit width

Upward base pressure W/B for unit width

Fig. 8

Its horizontal component $= R.\, d\alpha.\, \cos\alpha$

$\therefore$ Vertical force at the element

$$R d\alpha \cos\alpha\ W/B$$

Due to this vertical force the normal force developed at the element is δF_n

$$\text{where } \delta F_n = \frac{W}{B} R.\, d\alpha.\, \cos\alpha \cos\alpha$$

$$= \frac{WR}{B} \cos^2\alpha\ d\alpha$$

$$F_n = 2\int_0^\theta \frac{WR}{B}.\ \cos^2\alpha\ d\alpha$$

$$= \frac{2WR}{B}\int_0^\theta \frac{(1+\cos 2\alpha)}{2} d\alpha$$

$$= \frac{RW}{B}(\theta + \sin\theta.\cos\theta)$$

$$\sin\theta = \frac{B}{2R},\ \cos\theta = \frac{nD}{R},\ \tan\theta = \frac{B}{2nD};\ R = \sqrt{\frac{B^2+4n^2D^2}{4}}$$

$$F_n = \frac{W}{2}\sqrt{1+\frac{4n^2D^2}{B^2}}.\ \left[\tan^{-1}\frac{B}{2nD} + \frac{2nBD}{B^2+4n^2D^2}\right]$$

Moment of resistance of the base about the point of rotation

$$M_b = F_n \tan\phi\ R \qquad \dots \quad (1)$$

(ii) **For a circular base**

A multiplication factor of 0.6 is to be applied for the above expression of M_b in order to account for the surface of rupture being part of a sphere.

For both cases substituting the value "n" equal to 0.2D for the point of rotation in formula (1) above, the base resistance can be simplified and expressed in terms of B.

$$M_b = Q\, WB \tan\phi$$

where

B = width in the case of square and rectangular wells parallel to the direction of forces and diameter for circular wells.

Q = a constant, which depends upon the shape of well as well as the D/B ratio. Its values are given in Table 2 below for square or rectangular wells. A shape factor of 0.6 is to be multiplied for wells with circular base.

TABLE 2

D/B	0.5	1.0	1.5	2.0	2.5
Q	0.41	0.45	0.50	0.56	0.64

Note : The values of Q for intermediate D/B values in the above range may be linearly interpolated.

5.2. **Side Resisting Moment** (M_s)

The ultimate soil pressure distribution at the front and back faces of the well foundation is indicated in Fig. 9.

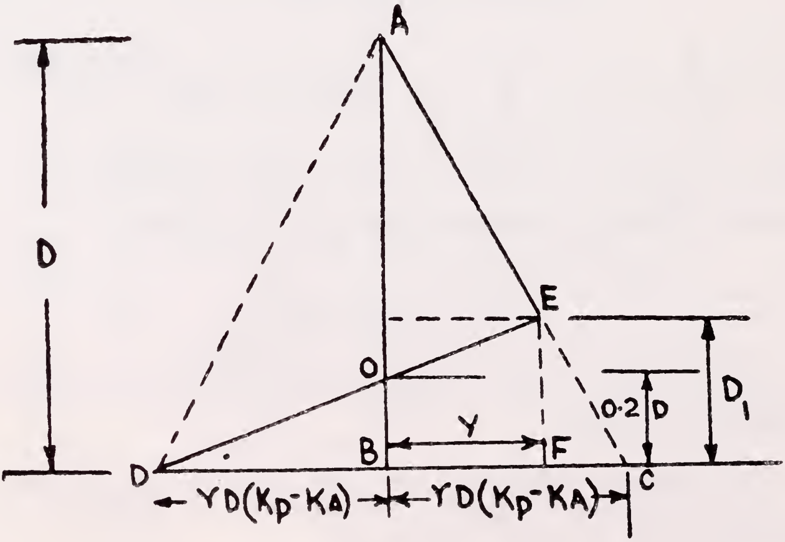


Fig. 9

The point of rotation is located at 0.2D above the base. The side resistance moment will then be calculated as follows :

Let, $\gamma D\,(K_P - K_A) = X = BC;\quad BF = Y$

From ΔDEF

$$\frac{D_1}{X+Y} = \frac{0.2D}{X} = \frac{D_1 - 0.2D}{Y}$$

or $$\frac{D}{X} = \frac{5D_1 - D}{Y} \qquad \ldots (1)$$

From Δ's ABC and CEF

$$\frac{D}{X} = \frac{D_1}{X-Y} = \frac{D-D_1}{Y} \qquad \ldots (2)$$

Equating (1) and (2)

$$\frac{5D_1 - D}{Y} = \frac{D - D_1}{Y}$$

or $6\,D_1 = 2D$:

where

$$D_1 = 1/3D \qquad \ldots (3)$$

Moment of side resistance about 'O' is the algebraic moments of Δ's ABC and DEC

$$= \tfrac{1}{2}\,D.X.\,\frac{2}{15}\,D + \tfrac{1}{2}\,\frac{D}{3}.\;\;2.X.\,\frac{4D}{45}$$

$$\frac{XD^2}{15} + \frac{4.X}{135}.\;D^2$$

$$= 13/135\;\;XD^2$$

$$= 0.096D^2.X$$

Say $= 0.1D^2X$

Substituting for X

$M_S = 0.1\,\gamma D^2\,(K_P - K_A)$ per unit width of well ;

For a width of L, $M_S = 0.1\,\gamma D^2\,(K_P - K_A).L$

5.3. Resisting moment due to friction on front and back faces (M_f)

Due to the passive pressure of soil as shown in Fig. 9, the frictional forces on the front and back faces of well will be acting in the vertical direction and will also produce resisting moment 'M_f'. For the purpose of this code, the effect of the active earth pressure perpendicular to the directions of applied forces is neglected. The resisting moment 'M_f' is calculated as follows :

The vertical pressure due to friction at any level is sin δ times the pressure at that level where δ is the angle of wall friction.

Total friction force/unit width = (Δ AOE + Δ BOD) sin δ

$\because$ $D_1 = D/3$

pressure at E $= \frac{2}{3} \gamma D (K_P - K_A)$

Area of Δ AOE $= \frac{2}{3} \gamma D (K_P - K_A) . \frac{0.8D}{2}$

$$= \frac{0.8}{3} \gamma D^2 (K_P - K_A)$$

Area of Δ BOD $= \frac{0.2D}{2} \gamma D \ (K_P - K_A)$

$$= 0.1 \gamma D^2 (K_P - K_A)$$

Total friction force/unit width

$$= \frac{1.1}{3} \gamma D^2 \ (K_P - K_A) \ \sin \delta$$

Moment about centre of rotation

(i) in case of rectangular wells for width L

$$M_f = \frac{1.1D}{3} \gamma D^2 \ (K_P - K_A) . \ \frac{B}{2} \sin \delta \times L$$

$$= \frac{0.55}{3} \gamma D^2 \ (K_P - K_A). \ B. \sin \delta \times L$$

$$= 0.183 \gamma (K_P - K_A) \ L.B.D^3 \ \sin \delta$$

$$\text{say } 0.180 \gamma (K_P - K_A) \ LBD^3 \ \sin \delta$$

(ii) In case of circular wells

$$\text{Lever arm} = \frac{B}{\pi}$$

$$\text{Therefore } M_f = \frac{1.1}{3}\,\gamma D^2\,(K_P - K_A).\ \frac{B}{\pi}\,L.\ \sin\delta$$

Since L = 0.9 B in case of circular well

$$M_f = \frac{0.33}{\pi}\,\gamma\,(K_P - K_A).\ B^2 D^2 \sin\delta$$

$$= 0.105\,\gamma\,(K_P - K_A)\,B^2 D^2 \sin\delta.$$

$$\text{say } 0.11\,\gamma\,(K_P - K_A)\,B^2 D^2 \sin\delta.$$

5.4. Total resisting moment of soil

Total resisting moment of soil M_r is given by

$$M_r = (M_b + M_s + M_f)$$

5.5. Factor of safety

A suitable safety factor has to be ensured taking into account the probable variation of different loads and their combinations. The variation of strength characteristic of the soil should also be accounted for in calculating the resisting moment given by the above expression. Putting it mathematically

$$\Sigma\, Y_i\ (\text{applied load or moment}) = \lambda\ (\text{soil resisting moment}) \qquad \text{..........} \quad (I)$$

where

Y_i = load factor for a particular load

λ = strength factor for the resistance of soil.

The passive resistance of the soil depends on the angle of internal friction for variation of which a reduction factor of 1.25 may be applied. Further to take into account the special nature of risk of failure of foundation, which is most important part of the bridge, another reduction factor of 1.15 may be applied. Hence the total coefficient applicable to the Right Hand Side of the above expression (I) will come to 0.7.

As regards the Left Hand Side of the expression, the variation of loads is described below :

(i) **Dead load** : The dead load being more or less a permanent load, a factor 1.1 would be sufficient for the variations in densities of materials and computational errors, etc.

(ii) **Live load** : Considering the effect of variation in IRC loading met with in bridges, it is adequate to adopt a factor of 1.6 for probable overloading with the combination of dead load only and 1.4 with other combinations except with wind or seismic. With either wind or seismic due to reduced probability of occurrence of maximum live load, a factor of 1.25 is considered adequate.

(iii) **Braking force, etc.** : These longitudinal loads will correspond to the coefficient adopted for live load.

Notes :

(1) The forces due to characteristic imposed deformations should be added, e.g., the horizontal load due to frictional resistance of the bearings may include the increase in dead and live load.

(2) For normal structures imposed temperature deformations of climatic origin and deformations due to creep and shrinkage can generally be neglected for the ultimate analysis. However, for statically indeterminate structures, the forces due to above causes should be considered. Similarly, the forces due to settlement of support have also to be taken into consideration.

(iv) **Water current force** : Due to possible error of 20 per cent in estimating the velocity, a factor of 1.4 may be adopted.

(v) **Buoyancy** : The effect of buoyancy in reducing the density of submerged masses is more or less a constant and can be taken as unity.

(vi) **Wind or seismic forces** : When the bridge is not covered by live load, a factor of 1.4 is considered adequate for wind or seismic forces. Due to less probability of combination with maximum live load, a reduced factor of 1.25 is adequate.

(vii) **Earth pressure on abutments** : To account for increased earth pressure resulting from either the density of soil being higher or

the angle of internal friction being lower than determined by tests for various reasons, a factor of 1.4 is considered adequate for computation of earth pressure.

Accordingly, the following combinations of load factors are obtained:

$$1.1D \quad \text{......... (1)}$$

$$1.1D + B + 1.4\,(W_C + E_P + W \text{ or } S) \quad \text{......... (2)}$$

$$1.1D + 1.6L \quad \text{......... (3)}$$

$$1.1D + B + 1.4\,(L + W_C + E_P) \quad \text{......... (4)}$$

$$1.1D + B + 1.25\,(L + W_C + E_P + W \text{ or } S) \quad \text{......... (5)}$$

where

D = dead load

L = live load including braking, etc.

B = buoyancy

W_C = water current force

E_P = earth pressure

W = wind force

S = seismic force

(viii) **Tilt and shift**: In the computation of applied moments, effects of moments due to tilt and shift of wells, if any, about the plane of rotation shall also be considered.

6. In order to ensure the factor of safety for ultimate resistance according to above concept, the total resistance moment (M_r) reduced by strength factor should be not less than the total applied moment (M) about the point of rotation for the appropriate combinations of applied loads enhanced by the factors given above, i.e., to say

$$0.7\,(M_b + M_s + M_f) \not< M$$